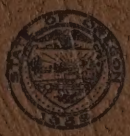
TECHNICAL REPORT No. 43-4

OREGON STATE HIGHWAY DEPARTMENT

R. H. BALDOCK, State Highway Engineer
C. B. McCULLOUGH, Assistant State Highway Engineer

The Geometric Design OF Highway Alignments and Highway Intersections



By

R. H. BALDOCK, State Highway Engineer

Published by

OREGON STATE HIGHWAY COMMISSION
SALEM

T. H. BANFIELD, Chairman
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STATE PRINTING DEPT.

TABLE OF CONTENTS

Section I—The Geometric Design of Highway Alignments with Relation to Selected Speeds

Page

Article 1—General Statement 1

Article 2—Horizontal Curvature 2

Article 3—Nonpassing Sight Distance 3

Article 4—Passing Sight Distance 3

Article 5—The Design of Horizontal Curves 4

Article 6—Stopping Sight Distance 6

Article 7—Measuring Sight Distance 10

Article 8—Design of Safe-passing Sight Distance 10

Article 9—Transition Curves 15

Article 10—Conclusions 18

Section II—Highway Intersections

Article 1—Introduction 19

Article 2—Minor Intersections 20
(a) Through Roads 20
(b) Speed Zones 22
(c) Flared Intersections 23

Article 3—Intersections at Grade with Traffic Signal Control 23
(a) Flashing Beacons 23
(b) Traffic-control Signals 24

Article 4—Intersections at Grade with Channelized Lanes and with or without Light-control Signals or Speed-change Lanes 27

Article 5—Intersections Providing Rotary Movement for Traffic Interchange 29

Article 6—Grade Separation Structures with Connecting Ramps 32

Article 7—Conclusion 34

SECTION I—THE GEOMETRIC DESIGN OF HIGHWAY ALIGNMENTS WITH RELATION TO SELECTED SPEEDS

Article 1—General Statement

The necessity for relocation of many highways has been brought about in the main by the rapid development of the internal combustion motor and the improved design of motor vehicles. For a long time, highway design did not keep pace with the innovations and changes in motor cars. The motor vehicle user is now demanding that highways be widened and straightened so as to permit higher speeds with less hazard. Many highways with a long life-expectancy in structures, pavements, and other facilities are now obsolete because their design does not permit higher speeds. Such highways suffer from functional obsolescence.

The fundamental control in the geometric design of highway alignments is that of the selected speed of traffic, and all factors of the design should be correlated to the selected standard of speed. Notwithstanding the almost certain possibility of further improvement of motor vehicles, permitting even greater speeds than those now attained, and the possibility of still further straightening of highways to decrease both horizontal and vertical curvature and thus to increase horizontal and vertical sight distance to meet these speeds, it seems certain that the nervous reaction time of the motor vehicle operator places a definite limit on speed.

The selection of the speed to be used as the basis for the design of highway alignments is controlled by the effect of centrifugal force on curves, by the horizontal and vertical sight distances required for stopping, by the necessity for frequent sections affording adequate passing sight distance, by the character and volume of traffic, by the cost of construction, and by the nervous reaction-perception time of the average motor vehicle operator. The geometric design for selected speeds is, therefore, controlled partially by formulae derived from the laws of mechanics and partially by empirical formulae based upon research.

The building and maintaining of highways must always be governed by the laws of economics. Highway systems must

be solvent, and the design of each road must be, to a great extent, governed by the revenue derived through motor vehicle fees and gasoline taxes. In consequence, construction cost is an effective control, and a design through heavy mountain sections is necessarily based upon a speed below the speed selected for level or rolling country. The design for a through, cross-state highway will, therefore, fall naturally into various speed zones, depending upon the economics of highway design. All factors of the design should be correlated with the standard speed selected for the zone in question.

In motorized highway transport, safety depends primarily upon the ability of the motor vehicle operator to see a sufficient distance ahead. This makes possible the control of the vehicle when encountering unexpected obstacles in its path and when passing overtaken vehicles. Inadequate sight distance is a very important physical characteristic which contributes quite largely to motor vehicle accidents.

Article 2—Horizontal Curvature

The speed of driving around curves is primarily controlled by the effect of centrifugal force. The radial acceleration is reduced by the superelevation or banking of curves and is counteracted to some extent by the frictional resistance between the tire and the pavement. The sudden application of centrifugal acceleration when leaving a tangent and entering a circular curve is alleviated through the use of transition or easement curves. The safe speed on a curve is sensed by the careful driver through the sway of the car and the force that pulls his body sideways. Motor vehicles driven around curves are subjected to a complicated system of forces, which in fact makes an exact mathematical analysis practically impossible.

For driving at low speeds on long-radius curves, it is possible to overcome centrifugal force entirely through the counteraction of gravity upon the mass of the car. The rate of superelevation is, however, limited by the slow traffic. In consequence, superelevation can give full compensation for only about 60 per cent of the high speed of modern traffic on account of the inability of slower moving vehicles to resist the action of gravity, which causes the vehicle to slide across the pavement under icy conditions.

In order to counteract the effect of centrifugal force for higher speeds, a frictional force is required, acting between the tire and the pavement, to supplement the force of gravity afforded by the superelevation of the curve. The frictional force is introduced by what is known as "slip angles," which angles may be defined as those angles between the horizontal diameter of the wheel and the tangent to the path of the wheel. Slip angles are introduced in both the front and rear wheels without conscious effort on the part of the driver.

The safe speed on curves is that speed at which the sway of the car is not uncomfortable. Speeds slightly greater tend to cause an uncomfortable sway, and, on a further increase in speed, the tires develop greater slip angles, and the increased speed on dry pavements causes a friction noise known as "tire squeal," which is an audible warning of an impending skid. There is, therefore, for all curves, a definite safe speed, and each curve should have a corresponding sight distance equal at least to the stopping distance for the designed speed.

Article 3—Nonpassing Sight Distance

In designing two-lane highways, there should be at all times a minimum sight distance, from the eye of the vehicle operator to the last portion of visible pavement, at least equal to the required stopping distance for the selected speed. This may be known as the "nonpassing" sight distance. The sight distance of both horizontal and vertical curves should at least equal the stopping distance required for the designed speed. Stopping sight distances vary from 200 feet at 35 miles per hour to 600 feet at 70 miles per hour as shown on the attached table.

Article 4—Passing Sight Distance

All highways have mixed traffic involving slowly moving heavy trucks and rapidly moving light vehicles. The cost of construction through even ordinary terrain is so great that it is not feasible nor economical to design a two-lane highway so as to make a safe-passing sight distance everywhere available.

The careful driver, on overtaking a slowly moving vehicle, usually waits until there is sufficient clear vision to afford an opportunity for safe passing. In the major portion of such instances, there is no opposing traffic, but the majority of the

operators are safety-minded and do not attempt to pass. In effect, they are avoiding "phantom" traffic. Safe-passing zones should be provided at frequent intervals in order not to encourage impatient drivers to attempt a passing maneuver which might result in hazard to themselves and to others. The passing sight distance required for 70-mile-an-hour safe speed is 2,600 feet. It would appear unwise to expend funds for sight distances materially greater than the amount required for safe stopping and less than the amount required for safe passing for the designed speed in the zone in question.

Article 5—The Design of Horizontal Curves

In designing horizontal highway curves, let us recall that well-known formula,

$$F = ma$$

in which F = force,
m = mass,
a = acceleration.

For centrifugal force, the acceleration is radial, and the formula becomes,

$$F = \frac{mv^2}{R} = \frac{wv^2}{gR},$$

in which v = velocity in feet per second,
w = weight of vehicle in pounds,
g = acceleration of gravity,
R = radius of curve in feet.

Up to a certain limit, depending upon the degree of curvature, centrifugal force is offset by the effect of the superelevation of the pavement and the side-skid friction between the tire and the pavement.

$$\therefore F = e + f = \frac{wv^2}{gR} = \frac{v^2}{gR},$$

wherein e = rate of superelevation,
f = friction factor,
w = unity.

Changing to V in miles per hour, the formula becomes,

$$V^2 = \sqrt{\frac{R(e+f)}{0.067}} = \sqrt{15R(e+f)}.$$

The rate of superelevation ranges from 0.02 foot per foot for the very light curves to 0.125 foot per foot for the heaviest curves. While many authorities believe that the higher factor for use on curves of 28 degrees or over is excessive, experience indicates that it is possible to use this amount in mixed traffic under snow and ice conditions provided the road surfaces are properly sanded. If they are not sanded, values of half that amount are found excessive. During the major portion of the year, the pavement is free from ice and snow, and the higher value permits a higher speed with greater safety.

Empirical data, together with experience in driving, indicates that the coefficient of side-skid friction should be taken as 0.17. The coefficient of side-skid friction must be sufficient to counteract that portion of the centrifugal force not compensated by superelevation. Inasmuch as the effect of centrifugal force acts not only on the car but also on the driver and other occupants of the motor vehicle, the coefficient used should not be so excessive as to cause an uncomfortable sway to the persons riding in the vehicle; for example, taking an extreme case and using a value of 0.5 for the coefficient of side-skid friction, which is a value that can be secured on most pavements, we may then determine the speed possible on say a 10-degree curve and the resultant side pull on the passenger in the car. Substituting the value of $e = 0.11$, $f = 0.5$, and $R = 530$ feet in the formula,

$$V^2 = \sqrt{15R(e+f)},$$

gives a speed of 70 miles per hour or 100 feet per second. Since centrifugal force equals

$$F = \frac{Wv^2}{gr},$$

the pull on a passenger weighing 150 pounds would equal 88 pounds. Superelevation will counteract 17 pounds, leaving an effective pull of 71 pounds. By reducing the friction factor to 0.17, the permissible speed equals 50 miles per hour and effec-

tive pull 30 pounds; likewise, on a 4-degree curve with a superelevation rate of 0.09 and a friction factor of 0.5, the permissible speed would be 112 miles per hour and the effective pull 72 pounds. By reducing the friction factor to 0.17, the permissible speed is 75 miles per hour and the effective pull on the passenger is 25 pounds. Since the maximum values above quoted are for speeds which may be deemed critical or overturning speeds, the reduction of the friction factor from 0.5 to 0.17 gives a factor of safety of 1.5 when taken from under the radical.

From the "Observations of Skidding Characteristics of Automobile Tires on Roadway Surfaces" by R. A. Moyer of Iowa State College, it was found that a side-skid coefficient of 0.3 (covering a wide range of speed, curvature, and driving practice) could be used when surfaces were so constructed as to provide a side-skid coefficient of not less than 0.6 at 30 miles per hour as measured with his test trailer; however, Moyer mentions that, when a coefficient of friction of 0.3 was required to counteract centrifugal force, the skill of the driver was taxed to the utmost. Extensive skidding tests made by the Oregon State Highway Department indicate that a side-skid factor of 0.17 under all conditions of pavement and speed normally encountered may, with perfect safety, be used. It is further felt that the design of selected speeds should be based upon factors which would not subject motor vehicle passengers to resist being pitched sideways by a force in excess of 30 pounds. A design contemplating a coefficient of side-skid friction of 0.17 minimizes danger and increases the safety materially.

Article 6—Stopping Sight Distance

The braking distance of a car is determined by the integration of the second derivative of the distance with respect to time, by which,

$$d = \frac{at^2}{2}, \qquad (1)$$

wherein d = braking distance in feet,
a = deceleration rate in feet per second per second,
t = time in seconds.

The stopping force or $F = f_2 = ma$ (2)

wherein f_2 = the coefficient of straight-skid friction between the tire and the pavement.

$$\therefore f_2 = ma = \frac{wa}{g},$$

assuming w = unity, $f_2 = \frac{a}{g}$ or $a = f_2 g$,

wherein g = acceleration of gravity.

Now $v = at$

in which v = the velocity in feet per second.

$$\therefore t = \frac{v}{a} \text{ and, since } a = f_2 g \text{ by substituting in (1),}$$

$$d = \frac{v^2}{64.4 f_2}. \quad (3)$$

Changing to V in miles per hour

$$d = \frac{V^2}{30 f_2} \quad (4)$$

In order to take into consideration the effect of gravity on grades, the formula becomes,

$$d = \frac{V^2}{30(f_2 \pm G)}.$$

In this formula, G = the gradient.

From mechanics, static friction is that friction which opposes motion when there is no slipping. The limiting friction is the value of the static friction when slipping impends. The coefficient of static friction is the ratio of the limiting friction to the normal reaction. In the case of the braking of a motor vehicle, the static coefficient of friction is the coefficient resulting from the application of the brakes gradually to a point wherein skidding in the line of travel impends.

Kinetic friction is that friction which opposes motion when one body is slipping on the surface of the other. In general, kinetic friction has a value less than that of limiting friction.

The coefficient of kinetic friction is the ratio of the kinetic friction to the normal reaction. In the braking of a motor vehicle, kinetic friction is manifest when sliding of the tire takes place in the line of travel. This is known as the "straight-skid" coefficient of friction.

Since the kinetic friction is a force stopping the vehicle which is equal to mass times acceleration, or, in this case, the deceleration, the coefficient of straight-skid friction (f_s) equals the rate of deceleration divided by the force of gravity, assuming a weight of unity; in other words, a deceleration rate of 16.1 feet per second equals a straight-skid coefficient of friction of 0.5. The coefficient of straight-skid friction may be defined as the ratio of the tangential force to the normal force.

The Bureau of Standards indicates that a comfortable rate of deceleration is 16.1 feet per second per second. The Motor Vehicle Department of the State of New Jersey assumes a comfortable rate of deceleration of 17.4 feet per second per second, which is equivalent to a coefficient of 0.54. Investigations made by the Oregon State Highway Department indicate that the rate of comfortable deceleration ranges between 12.9 and 16.1 feet per second per second, which corresponds to a rate equivalent to a friction factor of from 0.4 to 0.5. When stopping suddenly, a detached body in the car tends to be flung forward in proportion to its mass times the deceleration. A 150-pound passenger would have to resist a force of 60 pounds with a decelerating rate of 12.9 feet per second per second; likewise, a friction factor of 0.6, corresponding to a decelerating rate of 19.3 feet per second, would cause the occupant to resist a force of 90 pounds. A friction factor of 1.0, corresponding to a decelerating rate of 32.2 feet per second, would necessitate a resistance of 150 pounds, which is the weight of his own body. The driver is braced against the steering wheel, but, unless the occupants are strapped to the seat in a manner similar to that used in airplanes when landing, a fast rate of deceleration may become extremely hazardous. It is the belief of the writer that a force of 60 pounds on a 150-pound body can be resisted without discomfort. This corresponds to a straight-skid friction factor of 0.4 and to a deceleration of 12.9 feet per second per second. For the purpose of safety, this rate is recommended.

It will be noted that the maximum force exerted upon a 150-pound body in stopping is 60 pounds in comparison to the recom-

mended maximum of 30 pounds for the side-skid coefficient of friction; however, a passenger is better braced to resist a pull in line with the motion than he is to resist one normal to the line of motion.

Before the vehicle can be braked to a stop, there is a certain time required for the driver to perceive the need for stopping, known as the "perception time," and a certain period of time required to transmit the message from his brain through the nervous system to the muscles required to actuate the brake. This is known as the "reaction time." The sum of the two is known as the "perception-reaction time."

The report of the Massachusetts Highway Accident Survey, made in 1934 under the direction of the Massachusetts Institute of Technology, records the results of the determination of the perception-reaction time in which the stop light of the forward car was the action signal to the driver of the rear car. One hundred eighty individuals made 2,245 tests. The average perception-reaction time was 0.64 second; 5 per cent of the observations were in excess of one second; 20 per cent of the operators occasionally required at least one second; and 73 per cent always required more than 0.5 second.

Drivers were also tested under varying conditions, both in the laboratory and on the road, among the latter being tests in which the stop light was covered and the rear driver had to observe the slowing time of the forward vehicle. These tests indicated that a driver who showed a perception-reaction time of from 0.2 to 0.3 second under laboratory conditions might require 1.5 seconds to apply his brake under normal road conditions. These tests appear to justify the assumption that 1.5 second perception-reaction time is reasonable under normal road conditions.

The distance required for the perception-reaction time would be, therefore, when expressed in feet, equal to 2.2V, wherein V = speed in miles per hour. In consequence, the total stopping distance in feet becomes,

$$S = \frac{V^2}{30(f_2 \pm G)} + 2.2V.$$

In calculating stopping distance, assume G = —0.50.

Article 7—Measuring Sight Distance

As heretofore explained, the design of a highway based on selected speeds must provide sight distances equal to the stopping distance. In all cases, the sight distance from the driver's eye to the surface of the pavement on both horizontal and vertical curves should equal the stopping distance required for the designed speed on the curve and for the maximum speed permissible on the tangent in accordance with the posted maximum permissible speeds for the zone in question.

The available sight distance for a known radius of degree of curve may be computed by the following formula:

$$L = 200 \times \frac{\cos^{-1}\frac{R - m}{R}}{D},$$

wherein L = required sight distance,
R = radius of curvature in feet,
m = distance in feet to the side obstruction,
D = the degree of curvature.

This formula does not apply when the length of the circular curve is less than the sight distance or when the radius of curvature is not consistent throughout. When the length "L" (available sight distance) does not equal the stopping distance, the distance "m" must be increased. This formula is computed from quite obvious algebraic and trigonometric functions. Sight distances may also be scaled from the plans.

Article 8—Design of Safe-passing Sight Distance

The provision for frequent sections of straight highway alignment and long, sustained grades, affording requisite sight distance for safe passing, is necessary. Many people who instinctively sense the correct sight distance for safe passing have not the vaguest idea of the actual distance in feet. It is longer than most people realize. Like many other features in highway design, it is a function of mechanics and of human behavior.

Some time ago, the writer caused to be made a survey for the determination of the sight distance required for safe pass-

ing by the observation of the behavior of traffic. The critical conditions determining the sight distance required for safe passing may be assumed as follows:

1. The driver of passing Vehicle "A," traveling at the designed speed "V" of the highway, is forced by the presence of opposing traffic to slow down to the speed of Vehicle "B" ahead, which is assumed to be traveling at 10 miles per hour less than the designed speed of the highway.

2. The driver of Vehicle "A" requires a certain time, which will be termed the "perception-reaction time," to make up his mind, to react, and to start the passing maneuver. The distance intervening between Cars "A" and "B," at the instant that Car "A" cuts out of the right-hand traffic lane, may be termed the "cut-out" distance.

3. Vehicle "A" then accelerates to Speed "V." When Car "A" passes Car "B" and returns to the right-hand traffic lane, the distance between Cars "A" and "B," at the instant Car "A" cuts into the right-hand traffic lane, may be termed the "cut-in" distance.

4. During this passing maneuver, it is assumed that an imaginary, or possibly an actual car, traveling at Speed "V," is approaching in the opposite traffic lane, and it is further assumed that Car "C" reaches a point opposite Car "A" the instant after Car "A" returns to the safe position in the right-hand traffic lane.

5. The passing sight distance may be defined as the summation of the distances traveled by Cars "A" and "C" from the instant that the brain of the operator of Car "A" started to perceive, to react, and finally to motivate the muscles necessary to act upon the parts of the vehicle mechanism required to execute the passing maneuver until the two cars arrive at opposite and opposing positions in their respective traffic lanes.

The purpose of the field study was to determine by actual field test the distance required for passing Vehicle "A" to pass around the slower moving Vehicle "B." Phantom Car "C" was assumed to be approaching at the measured speed of Car "A," and the distance traveled by Car "C" was assumed to be the product of this speed and the observed time of the passage of Car "A" around Car "B".

The distance and the time required to pass around the overtaken vehicles were observed. Observations were made when the passed vehicle was traveling at speeds of 30, 40, and 50 miles per hour, respectively; likewise, the speed of the passing vehicle varied from 40 to 60 miles per hour. Under certain conditions, the passing vehicle slackened its speed to that of the vehicle ahead before attempting the passing maneuver on account of the presence of oncoming traffic or due to the lack of sufficient sight distance. In cases where the sight distance was adequate and no opposing traffic was visible, the passing vehicle found no necessity to slacken speed. The passing vehicle was, in all cases, operated by an unknown driver unaware of the fact that the tests were under way. The requisite data were secured by observers riding in automobiles moving in the stream of traffic. One automobile was introduced into the line of traffic and driven at constant speed. The distance between the passing vehicle and the vehicle being passed was observed when the passing maneuver started. This distance, termed the "cut-out" distance, was measured by gauging the width of the image between the front tires in the rear-view mirrow; likewise, the distance between the two vehicles after the passing maneuver had been completed, termed the "cut-in" distance, was determined by a scale mounted on the windshield. All passenger automobiles have approximately the same gauge, and, therefore, the distance from the outer edge of the tire to the inner edge of the opposite tire is approximately constant. The windshield and the rear-view mirror were calibrated by direct readings on parked cars over measured distances.

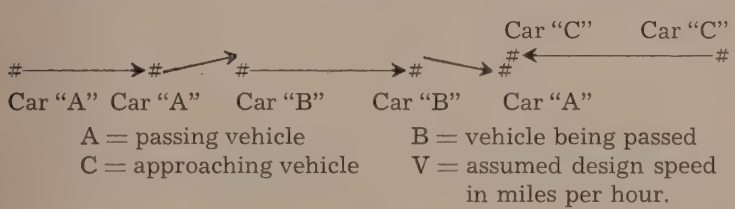
The second automobile, equipped with an odometer reading to hundredths of a mile, followed by a distance sufficient to permit any vehicle moving in the traffic stream to pass and to take its place between the two observing vehicles. When such a vehicle had taken its place in the line of traffic between the observing vehicles, the second testing vehicle was accelerated to the speed of the vehicle moving ahead. This speed was read and recorded by the observer. As the passing vehicle overtook the first testing vehicle and cut out from the line of traffic to start the passing maneuver, the driver of the second observing vehicle started a stop watch and noted the point on the road surface opposite the "cut-out" point. When the observ-

ing vehicle reached this point, the odometer was read and recorded by the observer.

As the passing vehicle assumed its place in the line of traffic ahead of the passed vehicle, the observer in the trailing vehicle shut off the stop watch and noted the point on the road surface where the passing maneuver was completed. The odometer was read and recorded by the observer when the observing vehicle reached this point. The difference between the odometer readings represented the total distance required to complete the passage. The time of passing was also recorded.

Observations closely simulating the critical conditions as hereinbefore defined were made and recorded. Following is a computation of theoretical conditions in comparison to actual observations of close similarity. The sum of the "cut-out" and "cut-in" distances was found to average 140 feet. The acceleration of motor vehicles of standard makes, as determined by another series of tests made by the Oregon State Highway Department, was found to average 2½ miles per hour per second. These values were used in the theoretical computations.

Sketch Illustrating Passing Maneuver



A = passing vehicle
B = vehicle being passed
C = approaching vehicle
V = assumed design speed in miles per hour.

Assume Vehicle "B" is proceeding at Speed "V-10"; assume Vehicle "C" is approaching at Speed "V"; assume Vehicle "A" proceeds at Speed "V-10" through distance "X," then accelerates to Speed "V," overtakes and passes Vehicle "B," and safely clears approaching Car "C."

Distance "X" equals distance traveled during perception-reaction time of driver of Vehicle "A," which time is assumed to be 1½ seconds for V-10 miles per hour $= 1.5 \times 1.47$ (V-10).

Distance "Y" equals safe clearance between Vehicles "A" and "B," assumed to be 70 feet.

Distance "Z" equals distance traveled by Vehicle "B" during time of passing maneuver, which equals 1.47 t(V-10).

Distance "K" equals distance traveled by approaching Car "C" during time of passing maneuver, which equals 1.47 V(t + 1.5).

Car "A" accelerates from Speed V-10 to Speed "V" at rate of 2.5 miles per hour per second in four seconds. Car "A" travels with relation to Car "B" while passing Car "B" a distance of 2Y, or 140 feet. Car "A" travels with relation to Car "B" while accelerating,

$$\frac{10 \times 1.47}{2} \times 4 = 29.4 \text{ feet.}$$

The total time "t" of accelerating and passing equals

$$4 + \frac{140 - 29.4}{10 \times 1.47} = 11.5 \text{ seconds.}$$

Total passing distance "d" $= X + 2Y + Z + K$
$= 1.5 \times 1.47\ (V\text{-}10) + 140 + 1.47\ t(V\text{-}10) + 1.47\ V(t + 1.5)$
$= 2.2V - 22 + 140 + 16.9V - 169 + 19.1V$
$= 38V - 50.$

The following table illustrates the relationship between the observed and computed passing distances under the conditions hereinbefore described.

Designed Speed	*Observed Passing Distance Required*	*Computed Passing Distance Required*	*Suggested Safe Values*
40	1200	1450	1500
50	1600	1850	1900
60	2100	2250	2300
70	--------	2600	2700

The above values given for passing distance are considered safe. Safe-passing sections should be designed at intervals of not to exceed one mile, if possible. Skilled drivers operating modern, high-powered cars can accelerate to a speed of from 20 to 30 miles per hour faster than the speed of the vehicle being passed, and they can cut in and cut out in shorter distances,

which may reduce the passing distance by half of the amount above shown. Such maneuvers are, however, of questionable safety and hardly possible for the average driver.

Article 9—Transition Curves

Transition curves are desirable to reduce centrifugal acceleration which is suddenly applied at the point of curve. The necessity for the use of transition or spiral curves increases as the speed is increased, since centrifugal acceleration varies as the cube of the speed. The State Highway Department of Oregon has used the transition curve for about seven years, and the results have been very satisfactory. Royal-Dawson in his "Elements of Curve Design" proposed the following transition formula:

$$L = \frac{3.155\ V^3}{R}$$

wherein L = length in feet of the transition curve,
V = velocity in miles per hour,
R = radius of the circular curve in feet.

The acceleration toward the center of the curve is $\frac{v^2}{R}$, and the rate of gain of acceleration is the acceleration divided by time, or $\frac{v^2}{R}$ divided by $\frac{L}{v}$; that is, $\frac{v^3}{LR}$. In other words, the acceleration gain is of $\frac{v^3}{LR}$ units per second per second in a second. If C is the maximum rate of acceleration that will pass unnoticed or, in other words, not cause a decided lurch of the car, then $C = \frac{v^3}{LR}$. Royal-Dawson, in the above transition formula, has assumed for C a value of one foot per second per second per second, or, in other words, the length of the spiral equals $\frac{v^3}{R}$ when v is in feet per second or equals $\frac{3.155\ V^3}{R}$

when V is in miles per hour. Moyer of Iowa State College agrees that the rate of change of acceleration may exceed 4 feet per second per second per second before an actual skidding condition is reached and that the value of 3 can be used without any decided discomfort. The latter value agrees with Oregon experience and has been adopted in its design; therefore, the length of the transition curve, in accordance with the formula adopted in Oregon, equals

$$L = \frac{1.0517\ V^3}{R}.$$

The table on Page 17 shows the sight distances required for various design speeds and various degrees of curvature and the amount of widening required when the limiting factor is the side slope of a 1:1 cut.

Table I

Minimum Standards for Design of Highway Alignments Depending upon Traffic Density, Character of Traffic, Assumed Designed Speed, and Topography

(The required sight distances are based on safe stopping distances for various speeds)

Highway Classification		Character of Terrain	Maximum Horizontal Curve Permitted	Width of Subgrade Including Ditches	Distance From Center Line to Side Obstruction 1:1 Slope	Additional Width Required On Inside Blind Curves	Horizontal Sight Distance		Minimum Vertical Sight Distance Required	Passing Sight Distance Required
Average Maximum Hourly Traffic Number Vehicles	Safe Speed m. p. h.						Required	Provided Without Additional Width		
			Degrees	Feet	Feet	Feet	Feet	Feet	Feet	Feet
	70	Level and rolling	3	46	27.5	3.0	600	562	600	2700
200-400	60	Foothills	6	46	27.5	2.0	475	459	475	2700
	50	Mountains	10	46	27.5	.....	350	356	350	2700
	70	Level and rolling	4	42	25.5	5.0	600	541	600	2700
100-200	60	Foothills	6	42	25.5	4.0	475	442	475	2700
	50	Mountains	10	42	25.5	1.0	350	344	350	2700
	70	Level and rolling	4	34	21.5	9.0	600	497	600	2700
50-100	55	Foothills	8	34	21.5	7.0	400	352	400	2700
	45	Mountains	12	34	21.5	2.0	300	288	300	2700
	60	Level and rolling	6	30	19.5	10.0	475	387	475	2700
15- 50	45	Foothills	12	30	19.5	4.0	300	274	300	2700
	35	Mountains	18	30	19.5	.....	200	224	200	2700
	45	Level and rolling	12	26	17.5	6.0	300	259	300	2700
5- 15	35	Foothills	18	26	17.5	.....	200	212	200	2700
	20	Mountains	56	26	17.5	.....	100	121	100	2700

The curvature used must provide the requisite horizontal sight distance for the maximum safe speed on the curves by widening the inside of blind curves as shown. The vertical sight distance on tangents should be such as to afford a safe speed of 70 miles per hour unless the designed speed of a particular highway is specifically given. The vertical sight distance on curves should be the requisite amount for the safe speed of the curve in question. The maximum curves should be used only where topography or other conditions of the terrain make the use of lighter curves too expensive. Both horizontal and vertical sight distances are the distances from the observer's eyes, assumed to be 4½ feet above the pavement grade, to the last surface of pavement visible in the opposite traffic lane. Where possible, the design for passing sight distance should be increased to 3,000 feet.

Article 10—Conclusions

Motor vehicle users are demanding that the highways be widened and straightened in order to permit faster driving speeds. Highway design has lagged behind the improvement in motor vehicles. Many highways with a long life-expectancy in structures, pavements, and other facilities are now obsolete because their design does not permit higher speeds with safety. Such highways suffer from functional obsolescence.

The fundamental control in the geometric design of highway alignments is that of the selected speed of traffic. All factors of the design should be correlated to the selected standard of speed. While it would appear that it is possible to build motor cars capable of attaining still higher speeds and it is likewise possible to design and construct highways permitting higher motor vehicle speeds, it would seem certain that the nervous reaction time of the motor vehicle operator places a definite limit upon speed.

The selection of the correct speed for the design depends upon:

1. The character and amount of traffic;
2. The cost of construction;
3. The effect of centrifugal force on curves;
4. The horizontal and vertical sight distances required for stopping;
5. The necessity for frequent sections affording adequate safe-passing sight distance;
6. The nervous perception-reaction time of the average motor vehicle operator.

The author proposes that highways be zoned for selected speeds, based upon the above criteria. The engineer should build safety into highways through correct design. The design involves the basic principles of mechanics, the findings of research of the action of motor vehicles at varying speeds, and of the behavior of the average driver under specific conditions.

SECTION II—HIGHWAY INTERSECTIONS

Article 1—Introduction

The subject to be discussed, highway intersections, covers a wide field. In general, highway intersections can be grouped in five broad classes listed as follows:

(a) *Minor intersections at grade;*
(b) *Intersections at grade with traffic signal control;*
(c) *Intersections at grade with channelized traffic lanes and with or without traffic signal control or speed-change lanes;*
(d) *Intersections at grade providing rotary movement for traffic interchange;*
(e) *Intersections providing for separation of opposing traffic streams by grade separation structures with or without connecting ramps.*

One of the intersecting highways usually carries a preponderance of traffic. We will define such highway as the "preference" highway; the other, as the "nonpreference" highway. Since few intersecting highways have grade separation structures, our major problem is that of highways intersecting at grade. Many of them are minor in importance, while others carry large traffic volumes.

It is difficult to cover a highly-involved subject such as highway intersections in a paper of this length, and, in consequence, only the broad, general principles can be touched upon. The correct solution of the many problems presented involves a mass of detail. No two intersections are exactly alike, and each requires special study. This discussion will merely touch upon the basic features of geometric design such as sight distance, grades, alignment, and placement of signs as affect the design of an intersectional area; likewise, time permits but limited consideration of such important phases as the assumed design speeds of each highway at the intersection, the dimensions of the assumed design vehicles, and the driving characteristics of the average operator.

Before any improvement is made at a highway intersection, a traffic survey should be conducted to determine the type and density of the traffic on both intersecting roads, the volume of

traffic constituting each of the turning movements, the alignment and grade of intersecting roads, and the angle of intersection; likewise, the physical conditions at the crossing, such as topography and culture which directly bear upon costs of construction and costs of right of way, are pertinent factors in choosing the type of intersectional design. An adequate design should provide accommodation for the type of vehicles likely to use the highway. The dimensions of maximum-sized vehicles using the intersection and the speed required for the traffic interchange are likewise determining factors in the intersectional design. It is not always feasible to design for the occasional passage of extremely large vehicles when such design materially increases the cost.

Article 2—Minor Intersections

Most of the intersections in the highway network of this country fall in the category of minor intersections at grade. In general, traffic on one or both intersecting roads is low in density. As a rule, the intersection is at right angles or nearly so. Intersections forming flat angles present a positive hazard because of the necessity for the driver to look over his shoulder to obtain a view of traffic approaching on the other road. Inasmuch as this requires some effort, many drivers fail to do it and collisions result.

Many intersections have impaired sight distance on one or both intersecting highways. Few attempts have been made to control speed at intersections, and the average motorist has a false sense of security because he seldom meets another vehicle approaching from one of the minor-traffic side roads. While available funds limit to a major extent the corrective measures requisite to improve materially such conditions, a few of them will be listed in the order of the relative importance of the intersections.

(a) *Through Roads.* It would appear that it is quite hazardous to permit traffic to move through intersections at grade totally uncontrolled. The selection of the preference highway as a "through" highway is a decided improvement. Under such conditions, all traffic entering upon or crossing the preference highway must come to a full stop and delay entering or crossing until safe clearance is assured. The stop sign should be located

near the place where it is desired that vehicles come to a stop. The stop should be made close enough to the intersecting highway to permit as unrestricted a view as possible but far enough back from the traveled way to avoid conflict with vehicles turning into the side road. An advance warning sign such as "Prepare to Stop" should be placed on the nonpreference road sufficiently in advance of the point of stopping to permit vehicles to decelerate speed and come to a complete halt before reaching that point.

An advance caution sign should, under certain conditions, be placed on the preference road sufficiently in advance of the intersection to permit the operator of the vehicle on the preference highway to stop, if necessary. When the intersection is clearly visible for a long distance and the speed on the preference highway is not too great, advance warning signs are usually omitted. The determination of the distance from the advance warning sign to a point just clear of the intersecting highway should be at least equal to the stopping distance of the approaching vehicle.

The stopping distance is the sum of the distance traveled by the vehicle during the reaction time of the motor vehicle operator and the distance traveled during the braking time of the vehicle. Perception time is that time required by the average motor vehicle operator to size up a situation. Ignoring inattention, which cannot be considered in design, perception time in this case is the time required to read and identify the sign or to see the intersection ahead, so, apprised either by the advance warning sign or by the view of the cross road, the driver on the preference road further observes the intersection provided no obstruction limits vision. If no vehicles are in sight, speed may be only slackened slightly, but the operator should be on the alert. On the other hand, if the vision is obstructed, the prudent driver materially slackens speed when he sees the advance warning sign so that an emergency stop can be made, if necessary, to avoid collision. Impaired sight distance at intersections warrants speed-control signs, which subject will be discussed presently.

The driver on the nonpreference road, on perceiving the advance warning sign, should slacken speed and prepare to stop, regardless of sight conditions. Since the perception time

should not require any travel distance beyond the advance warning sign (assuming the lettering on the sign is such that it can be read before the sign is reached), we need only consider reaction time. Reaction time is defined as the time interval required, after perception, for the motor vehicle operator to start braking the vehicle. Tests indicate this varies between one-half and one second. Most investigators have used a safe value of one second.

The distance traveled by a vehicle during the reaction time and the braking time is a function of the speed of the vehicle before braking starts. Assuming all vehicles approaching the intersection are traveling at the assumed design speed of the highway, the posting distances for caution or advance warning signs are as given in the table which follows. Since observations on many highways show that the average vehicle is traveling at only 0.7 of the assumed design speed of the highway, an additional factor of safety is introduced.

Posting Distances for Caution Signs

Highway design speed (V), m. p. h.	30	40	50	60	70
Reaction distance (1.47 V), feet	44	59	73	88	103
Braking distance (0.083 V^2), feet	75	133	208	300	408
Calculated stopping distance, feet	119	192	281	388	511
Suggested posting distance, feet	100	200	300	400	500

The deceleration rate of the braked vehicle is assumed as 12.9 miles per hour per second, which corresponds to a friction factor of 0.4, which friction factor is readily available on almost all surfaces except when covered by ice. A more detailed discussion of design speed with relation to stopping distance is given in the brochure of the American Association of State Highway Officials entitled "A Policy on Sight Distance for Highways."

Occasionally the driver of a vehicle on a nonpreference road does not stop the vehicle; however, as far as Oregon is concerned, where "through highways" have been used for many years, this is a rare occurrence. It is, however, certain that the "through highway" principle does not relieve either motorist from exercising due caution and prudence.

(b) *Speed Zones.* Attempts have been made in many states to zone highways in relation to design speeds based upon sight

distance and other features of geometric design. This appears to be a reasonable approach to a perplexing problem, and it is believed that it will finally be universally adopted. With proper enforcement, the accident rate should materially decrease.

Speed zones are particularly needed at intersections with impaired sight distance. The ordinary "slow" signs which are usually placed near such intersections are not adequate to meet many situations and are not properly observed by many motorists. Speed zones at intersections can be regulated by traffic-control signs which will be discussed under the heading of "Intersections at Grade with Traffic Signal Control."

(c) *Flared Intersections.* When the paved area on the traveled way is not widened on either highway at an intersection, it is impossible to make some of the turning movements without encroaching upon the opposing traffic lane. On the contrary, the use of large "Y" intersections without provision for an acceleration or speed-change lane to enter the preference highway has the disadvantage of requiring a stop on an acute angle, thereby limiting the operator's vision. As a rule, it is better to flare an intersection by widening the paved area on both roads so that the turning movements can take place without encroachment on the opposing lane. In general, the curves should be spiraled, or at least three-centered, and have radii of sufficient length to permit a turning movement of the maximum-dimension vehicle which may use the intersection. The length of radii of curves is also in direct proportion to the desired speed of the turning movement. On Pages 10-27 of the brochure published by the American Association of State Highway Officials entitled "A Policy on Intersections at Grade," detailed information with reference to this problem is given.

Article 3—Intersections at Grade with Traffic Signal Control

(a) *Flashing Beacons.* When the traffic on intersecting highways, increases, warning flashing beacon signals are often installed. Such signals continually flash the amber caution color to the preference highway and the red stop color to the nonpreference highway. Such signals thus only partially control intersectional traffic, but caution the operator on the preference highway of the presence of an important intersection ahead. These protective devices are quite effective pro-

vided they are not installed indiscriminately. Too many "blinker lights" cause the motorist on the preference highway to disregard them. When they are placed at particularly hazardous intersections, a fair degree of observance is secured.

The warrant for their installation is largely a matter of opinion based upon experience. In an attempt to standardize this practice and to approach some uniformity in the installation of flashing beacons, the Oregon State Highway Department has sought some method to determine the particular need for an installation of this kind. There appears to be no hard and fast rule. Traffic volume, sight distance, and traffic speed are all factors, while the accident record, indicating the character and severity of the accidents, is quite important.

(b) *Traffic-control Signals.* When the traffic on the intersecting highways reaches the point wherein it becomes difficult for the vehicles on the nonpreference highway to enter into or cross through the preference highway, further control becomes necessary.

Traffic signal control is familiar to all motorists in this country. Two general types of signal control are used: the fixed-time signals and the traffic-actuated type. There are several variations in both. A detailed description of each type follows.

Fixed-time Signals. Traffic-control signals are usually operated on the fixed-time principal in which the time intervals allotted to each movement are controlled by an electrically-operated timing device or "controller." Traffic studies indicate the correct time intervals for each movement. The time phases may vary with the time of the day, based upon observed traffic-volume fluctuations over an extended time. When careful traffic surveys are made, excellent results are attained. Generally the signal ceases to operate on a fixed-time basis after certain hours on certain days of the week and merely functions as a flashing beacon thereafter until a certain time in the morning.

Left-turning movements are the bane of intersections at grade. If the left turns are numerous, a separate time interval may be given for the left-turning movement, the red light holding the opposing traffic streams until the movement has

taken place. Unless this is done, a left-turning movement at certain intersections becomes quite hazardous and causes many intersectional accidents. As the traffic further increases, the delay caused by the time interval for the left-turning movement causes long strings of vehicles to wait for the movement to take place and may actually block traffic at intersections on either side of the intersection in question. When this condition materializes, it is necessary to eliminate left-turning movements entirely at the intersection.

Quite often, fixed-time signals installed at several intersections on a street are further controlled by a master controller, which regulates the local controller, so that traffic on the preference highway may proceed uniformly at a predetermined speed rate without stopping at the intersections. Under such conditions, the signals serve to control speed. The introduction of the separate left-turn time interval usually prevents the use of progressive time signals.

Traffic-actuated Signal Control. When the traffic on the preference road materially exceeds that of the nonpreference road, traffic-actuated signals are preferable. Two types of traffic-actuated signals are used—the semi-actuated and the full-actuated types. In the case of the semi-actuated type, detectors are installed in the right-hand lane of the intersecting nonpreference highway at a predetermined distance. When a vehicle passes over the detector, the current is broken, and the traffic-control signal is actuated so that, after a certain time interval, traffic on the nonpreference road gets the green light—the signal to proceed. The signals remain so fixed for a definite time, sufficient to permit the entering vehicle to complete its movement, after which the red light appears on the non-preference highway and the green light returns to the preference highway. A definite and appreciable time interval must elapse before the green light again appears on the nonpreference road, as otherwise the traffic on the preference road may be materially delayed and traffic jams may develop. The time phases vary with the character and amount of traffic as disclosed by the traffic surveys.

The full-actuated type of signal control involves the placement of traffic detectors in all lanes approaching the intersection. When traffic is fairly uniform on both intersecting high-

ways, the signal control affords time intervals for the various traffic movements approximately equal to those provided by fixed-time signals. When the traffic fluctuations become too irregular, a comptometer may be installed to regulate the traffic flow. A description of this type of signal control will be given presently.

Full-actuated traffic signals may be used to control the speed of approaching motor vehicles on both intersecting highways. Under such circumstances, the red stop light normally appears in all four directions. When a vehicle approaches on either highway at a speed not in excess of the posted speed, the green light appears in the one direction only. The green light should appear when the approaching vehicle is approximately 50 feet in front of the signal. All other traffic is held until the intersection maneuver is completed. When the vehicle approaches at a speed greater than the posted speed, the red light does not change to green by the time the vehicle reaches the intersection; therefore, the vehicle operator must stop the vehicle and wait until the light changes to green. This soon teaches the operator to refrain from driving at a speed greater than the posted speed. When a detector on either intersecting highway is actuated, the first vehicle to arrive at a detector has clearance through the intersection. Thus this device will protect left-turn movements as well as through movements and likewise tend to control speeds approaching the intersection from both directions. When there are too many vehicles entering the intersection from the nonpreference road, the signal may be set so that a chosen interval of time must elapse before the clear signal will show again on the nonpreference road.

More recent developments have brought about a type of traffic-control device, of the full-actuated type, which actually makes its own traffic count. This interesting robot is actuated by a comptometer which adds the traffic passing over detectors installed in both the preference and nonpreference highways and regulates the time intervals of the signal control in accordance with the fluctuation in the traffic movements. This surprising gadget has not only the power of calculation based upon the actual phenomena observed, but has, in addition, the power of memory. It does not forget that it has passed a vehicle through to the intersection and permits the required time

interval to elapse before the lights change. In the meantime, the red light prevents another entry.

Many intersections can be quite well policed by the correct type of traffic signal. The use of such signals is a very important phase of highway intersection design. As in the case of flashing beacons, judgment and experience are quite necessary in the choice of their location. Traffic-control signals, when used indiscriminately, constitute a real barrier to free traffic movement and arouse the ire of the motorist. When the motor vehicle operator approaches a red light at a free intersection, the error in either installation or operation is obvious. Moreover, the cost of traffic-control signals is from fifteen to twenty times as much as flashing-beacon caution signals; therefore, the warrant for the use of the more expensive devices should be carefully studied. The Oregon State Highway Department, in general, does not advocate the use of traffic-control signals unless the traffic on the preference road averages at least 750 vehicles per hour and the hourly traffic on the nonpreference road approaches approximately 250 vehicles for eight consecutive hours.

Article 4—Intersections at Grade with Channelized Lanes and with or without Light-control Signals or Speed-change Lanes

The use of wide, flared, paved intersections may cause confusion, particularly as the traffic increases. The correct position for the vehicles for each of the various turning movements must be well defined; for instance, the presence of a vehicle in the right lane when the driver intends to execute a left-turning movement snarls all the traffic at the intersection. Painted lanes are an aid, particularly when the words "left-turn lane" or "right-turn lane" are painted on the pavement in the proper location.

A better way is to install traffic dividers advising traffic of the proper lanes by means of large overhanging signs directly over the painted traffic lanes before the physical traffic dividers are encountered. The traffic dividers widen into islands at certain "dead zones" in the paved area. These serve as strategic locations for the installation of information and direction signs and also as a refuge for pedestrians crossing the intersection. A wide, paved intersection without islands is a

deathtrap for pedestrians. It is impossible to watch out for traffic in all directions. The use of traffic islands requires the pedestrian to watch in one direction only, since the traffic movements are made by following one-direction routes.

Left-turn movements at highway intersections at grade are the source of many traffic accidents. Provision for a left-turn lane has been found to give excellent results. The left-turn lane should be built in the preference highway and can be used in either a two- or a four-lane road. The opposing lanes are spread apart at the intersection, and the resultant segment serves as a haven of refuge for the vehicle desiring to make a left-turning movement. The use of a pavement of a contrasting color and texture is preferable, and an overhead lighted sign is recommended to inform the motor vehicle operator of the presence of the left-turn lane. Curbs or other barrier devices may be used to mark the "slot" for the left-turn lane. The driver can wait in the lane without the hazard of conflict with traffic until the intersection is clear. The public soon grasps the significance of the plan and readily makes use of the improved and safer facility.

By means of speed-change lanes adjacent to the regular traffic lanes, traffic may decelerate to turn off either highway or accelerate to join the traffic streams on either highway without stopping. The change in speed required is a function of the design of the curves at the intersection. If possible, the radii of the curves should be such as to permit speeds not less than 40 per cent of the design speed of the preference road. Cross traffic may require traffic-signal control. In the case of "T" intersections where the preference road is a four-lane highway and where traffic signal control is used, the outer lane of the four-lane highway can be separated from the inner lane by a traffic divider. Green arrow lights can then be installed, permitting traffic to proceed on the outer lane at all times, regardless of the signal control operation. The inner lane should be marked by overhead signs for left-turn movements; the outer lane, for through movements. This type of channelization materially expedites movements at "T" intersections. Many times it is found to be an advantage to widen a two-lane preference road to a four-lane facility at an important intersection to facilitate turning movements and minimize accidents. Chan-

nelized intersections can be used to advantage when the traffic is not great enough to justify either traffic signal control or speed-change lanes. In even minor intersections, island dividers to separate traffic can be used to great advantage.

Article 5—Intersections Providing Rotary Movement for Traffic Interchange

Another type of intersectional design between the types previously outlined and grade separation structures is known as a rotary intersection. Such intersections have been referred to as "traffic circles." This, however, is a misnomer, because the design of the central island is usually of various shapes such as rounded squares, diamonds, ovals, polygons, and other combinations of unsymmetrical shapes.

Essentially, a rotary is a one-way, closed-circuit road in which all traffic converging on a given point is directed and from which traffic emerges into the radial roadways. It might be described as a series of one-way roads joined smoothly to form a continuous one-way road connecting entrances to and exits from several roadways.

The path followed by a motor vehicle through a rotary intersection is a counter-clockwise, one-way movement around a central island. The crossing of vehicles is accomplished at small angles, resulting in a weaving type of movement. There has been a definite trend toward larger dimension rotary intersections affording easier curves and permitting higher travel speeds. The desirability of built-in, channelized control to induce the traffic to follow a desired route is generally recognized. The earlier designs were symmetrical in shape, but the symmetrical designs did not accommodate normal motor vehicle movement, and this resulted in the presence of sections of "dead" pavement with a resultant constriction of traffic at other points. Years of study have indicated a choice of design based upon actual traffic behavior so that the vehicle movements are simple and natural.

The increase of weaving length in rotary design serves to facilitate traffic flow even to a greater extent than the addition of one or more traffic lanes. Longer weaving areas enable traffic to weave at higher speeds crossing at smaller angles. The resultant range in weaving length between the minimum and

maximum suggested for selected design speeds varies from 150 feet for 25 miles per hour to 360 feet for a 40-mile-per-hour design; likewise, the minimum radius of a curve around the central island varies from 85 feet for 25-mile-per-hour design speed to 340 feet for 40-mile-per-hour design speed. The design of rotary intersections is very complicated and should not be attempted without the observation of movements in existing rotaries operating in this country and a consultation with their designers.

Many times rotary principles can be used successfully for minor movements, whereas the through crossing movements are taken care of by grade separations. Recent experience seems to indicate the feasibility of this plan. The earlier rotaries were mainly located in urban areas with heavy volumes of slowly moving traffic. This was at a time when motor vehicular speed was comparatively low, and the rotaries worked very satisfactorily with speeds of 15 to 25 miles per hour. It is the opinion of the author that rotary intersections effecting traffic interchange between roads designed for speeds of 50, 60, and 70 miles per hour will never be satisfactory; however, there are many possibilities in using the combined rotary and grade separation principles. The advantages of rotary design are:

1. A regimented traffic flow is provided by the rotary, one-way movement.

2. In rotary traffic, the flow weaving of traffic replaces the usual angular crossings inherent in ordinary intersections at grade. There is no direct conflict in direction of movement, all the traffic lanes merging or leaving at small angles. The effect of the small angle merging and the relatively slow speed rotary movement reduces the possibility of serious collision and accident loss. The accidents are for the most part dented fenders and others of a minor nature. There is little likelihood of serious collisions in a rotary movement.

3. Left turns can be made as readily as right turns or straight through movements.

4. A rotary can accommodate a comparatively large volume of traffic. It will carry a large volume of traffic faster through the intersection than the same volume of traffic through an intersection with traffic signal control, but smaller volumes

of traffic will move faster through an intersection with signal lights. For the most part, the average motorist prefers the rotary intersection rather than one with signal control because of the inherent dislike of the motorist to stop and start. In general, the rotary costs less to construct than a highway grade separation with properly designed ramps unless the topography particularly favors a grade separation design.

Disadvantages of rotary intersections:

1. A relatively large area is required for the use of rotaries; and, in congested areas, the cost of right of way may be, and quite often is, prohibitive.

2. Topographic conditions may make it undesirable to use rotaries at many junctions. Large level areas are required for a properly-designed development. Rotaries are not readily adaptable to mixed traffic, including pedestrians and bicycles. They are more adaptable to traffic that is uniform in character. The presence of heterogeneous traffic requires the installation of traffic-control signals to properly care for pedestrian traffic. For this reason, many of the benefits inherent to the type are lost.

3. The rotaries must be extremely large when used on high-speed roads; otherwise, traffic is required to slow down too much. Large, extensive rotary intersections often confuse drivers not familiar with this type of traffic interchange. The confusion leads timid motorists to stop or swerve, thereby causing accidents; likewise, strangers may become so confused that they may go entirely around the rotary. Improvement in lighting, signing, channelization, and the more extensive use of rotaries tend to minimize such confusion. Many people believe that the reduction in accident loss through the use of rotaries is debatable. The consensus appears to be that the accidents are reduced in seriousness but not greatly in number.

4. Rotary designs are not altogether a satisfactory treatment when roads intersect at acute angles because of the short weaving distances provided and the undesirable large crossing angles necessitated.

5. A rotary is not readily adapted to stage development.

In 1942, there was published by the American Association of State Highway Officials a "Policy on Rotary Intersections."

A number of illustrations of existing rotaries are given in this brochure, together with a wealth of discussion of design speeds, interweaving lengths, curves, geometric shapes, sight distance, roadway widths, cross slopes, curbs, islands, lighting, traffic-control signals, and special adaptations of rotary intersections.

Article 6—Grade Separation Structures with Connecting Ramps

It is obvious to anyone that intersectional accidents can be eliminated by separating the grades of the cross streams of traffic. If the mere separation of cross streams of traffic would accommodate all the traffic movements, the problem would be quite simple and the cost relatively nominal; however, it is usually necessary to effect an interchange of traffic between the cross highways, which sometimes makes a very difficult and costly problem, particularly when several highways intersect near the same place.

It is certain that the volume of traffic that can be passed through an intersection will approach the total capacity of the two intersecting highways through the provision of roadways at different levels so as to enable both through streams of traffic to flow without interruption.

When appropriate connecting roadways for the interchange of traffic between the intersecting separated highways are built in such manner as to eliminate cross-traffic movements, the ideal situation is realized. Separations can be made by passing the preference highway either under or over the nonpreference road. Such structures are known as "undercrossings" and "overcrossings." A definition of the terms used in this discussion of the conventional types of grade separation structures that have been used in this country is given as follows:

Ramp: A ramp may be defined as a connecting roadway between two intersecting highways at a grade separation. It may consist of 2 one-way roadways, physically separated throughout, referred to respectively as the "inner loop" and the "outer connection," their distinction being obvious. A ramp may be curved, diagonal, or parallel with respect to the highways.

One-way Ramp: This is a ramp for traffic in one direction only.

Directional Interchange: This is a form of ramp, generally one-way, which does not deviate greatly from the intended direction of travel. The inner loop for left-turning movements is avoided.

Cloverleaf: A cloverleaf is a grade separation with ramps in all four quadrants.

Partial Cloverleaf: The partial cloverleaf is a grade separation with ramps in either one, two, or three quadrants.

Trumpet: A trumpet is a grade separation at a "T" intersection, generally with three ramp connections.

"Y" Grade Separation: This is a "Y" intersection where two of the left-turning movements are separated. No ramps are involved, the road and connections accommodating the turns.

Braided Intersections: An intersection involving more than one grade separation structure or more than two levels of traffic, with directional interchange between roads. Essentially, braided intersections are combinations of individual grade separations and their ramps. Directional interchanges are provided so that the procedure for the desired turning maneuver is natural and obvious.

The most conventional type of grade separation that has been used is either the full or partial cloverleaf design. There is, however, a fundamental defect in the cloverleaf principle in that it confuses motorists wishing to make proper connection with the intersecting highway. All left-turn movements are made by two successive right turn movements. This is quite confusing to a motor vehicle operator who is not acquainted with the cloverleaf type of traffic interchange in general and with the case at hand in particular.

Large, lighted signs overhanging the traffic lanes are a distinct aid in properly directing travel through a cloverleaf intersection. An indication of the proper lane to weave into before the intersection is reached is quite helpful. The proper signing of such an intersection is in itself a major study and requires a wide knowledge of traffic behavior.

A braided intersection provides a more effective traffic interchange in that the correct traffic movements are obvious. When an operator of a motor vehicle, on approaching the intersection, desires to go to a certain destination, the directional

interchange proceeds in the proper direction. When the destination is to the left, the interchange facility passes either under or over the highway upon which the vehicle has been traveling. It is, of course, necessary to enter the correct interchange lane before reaching the point of diversion. An overhead, lighted sign should, in this case, be placed over the extreme right-hand lane. The sign should name the next important town to which the interchange lane leads. In a short time, the lane gradually turns off the highway, widens to two lanes, and then passes above or below the highway proper.

Braided intersections generally involve three or four traffic levels. Their design is determined primarily by the topography. While it is usually practical to attain a directional interchange in one or two directions, the addition of more may make a solution quite expensive if not impossible. Many times two directional interchanges will suffice, and, if necessary, the minor traffic turning movement can be cared for by methods which are less obvious as to direction but much less expensive.

As previously stated, grade separation structures combined with a rotary intersection design may afford a practical solution. The major traffic movements proceed without hindrance, the minor traffic movements by the rotary principle.

The warrant for the expenditure of the funds required to eliminate cross highway traffic at grade is largely a function of the delay and expense suffered by traffic. The evaluation of the benefits caused by the saving in time and saving in motor vehicle operating costs is worthy of special study and investigation. The results of some study on this problem, by this and other states, are given in "Oregon Technical Bulletin No. 7," and a further report will be made in "Oregon Technical Bulletin No. 16," as soon as it is published. It is the thought of the author that the elimination of cross traffic at grade on many highway intersections in this country will, upon investigation, prove to be economically feasible.

Article 7—Conclusion

No attempt has been made to show sketches of the types of intersections discussed because of the time required to properly present them. Rather, an attempt has been made to create word pictures, depicting the standard types of intersec-

tions which can be readily recognized. Reference has been made several times to the brochures on rotary movements and grade-separation design which have been published by the American Association of State Highway Officials. These brochures were prepared by the Special Design Committee of this association, of which the author is a member. Reference to these publications, together with a study of similar booklets now available on sight distance and intersections at grade, will provide further detailed information for those who may be interested.

Proper design of highway intersections is one of the more important phases of highway engineering. The solution of each problem, based upon the best design within the permissible expenditure of funds, requires a thorough knowledge of the principles of geometric design and of traffic behavior. When improvements are made at the intersections based upon the solutions of the many and varied problems involved, a major step will be taken toward the minimizing of highway accidents; likewise, progress will be made in expediting traffic flow, which is, after all, the main objective of the Highway Engineer.

PUBLICATIONS OF THE OREGON STATE HIGHWAY COMMISSION

Technical Bulletins

No. 1. Loading Tests on a New Composite-type Short-span Highway Bridge Combining Concrete and Timber in Flexure, by Baldock and McCullough. 1933.

No. 2.* Application of Freyssinet Method of Concrete Arch Construction to the Rogue River Bridge in Oregon, by Gemeny and McCullough. 1933.

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No. 15. The Experimental Verification of Theory for Suspension Bridge Analysis, by McCullough, Paxson, and Rosecrans. 1942.

Technical Reports

Report of the Interim Committee appointed to study the Motor Transportation Act, by the Interim Committee. 1937.

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No. 38-1A. Addenda to an Inventory of City Streets in Oregon, by Beakey, Van Scoy, and Walton. 1938.

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No. 38-5A. Annual Daily Traffic Density Tables of the Rural Traffic Survey, State-wide Highway Planning Survey (report accompanied by traffic station maps—price $5.00), by Beakey, Glenn, and Manning. 1938.

No. 38-6. Urban Traffic Survey (Part I and Part II), by Beakey, Glenn, and Manning. 1938.

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No. 40-3. Data Supplementary to Interim Committee Report of January 1, 1937, by Baldock, Beakey, and Myers. 1940.

No. 41-1. Traffic Density Tables for 1940, by Beakey and Manning. 1941.

No. 42-1. Traffic Density Tables for 1941, by Beakey and Manning. 1942.

No. 42-2. Data Supplementary to Technical Bulletin No. 10, by McCullough and Myers. 1942.

No. 43-1. Manual of Standard Practice for Paving Inspectors, by Finkbiner and O'Neil. 1943.

No. 43-2. Manual of Standard Practice for Sampling Construction Materials, by Finkbiner. 1943.

No. 43-3. Traffic Density Tables for 1942, by Crandall and Stein. 1943.

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